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DESIGN OF SKY TOWER

NEW ZEALAND'S TALLEST STRUCTURE

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INTRODUCTION

Sky Tower is part of the Sky City Development project presently under construction in Auckland City. The development includes a casino, hotel, conference, retail and parking facilities and Sky Tower. Design of Sky Tower commenced in May 1994 and construction is expected to be complete in mid 1997.

STRUCTURAL ARRANGEMENT

Figure 1 provides a cross section through the Sky Tower with overall dimensions and main features noted.

The tower springs from a foundation the top of which is located approximately 8 m below street level.

The foundation comprises a 2.5 m thick 24.5 m diameter reinforced concrete pad which bears directly on siltstone rock. Bearing is supplemented by sixteen 2.0 m diameter reinforced concrete grooved piles which are arranged around the perimeter of the pad and penetrate 12 m into the siltstone to below the level of the adjacent car park. A reinforced concrete circular ring beam interconnects the tops of the piles providing lateral restraint to the tops of the piles independent of the pad foundation and confining the rock within. The main pile reinforcement is not fully anchored into the pad. Instead a smaller, closely bound, reinforcement cage is provided on the centreline of each pile which is fully anchored into both the pad and the pile. In the event of severe lateral overload (such as might occur in a very severe and unexpected earthquake) this reinforcement cage is intended to *yield* allowing the tower to rock and thus protecting the piles against an uplift failure.

The main structural element of the tower is the reinforced concrete shaft which is 225.6 m in height. This shaft is tubular with a constant external diameter of 12 m and wall thickness ranging from 500 mm at the base to 350 mm over the upper levels. Within the shaft there are a number of internal walls including a 200 mm cross wall, slightly offset from a shaft diameter, and 150 mm walls forming the enclosures to the lift shafts and stairwells. These walls are cast integral with the main shaft.

Eight raking reinforced concrete legs stiffen the lower position of the shaft and are joined to the shaft via a prestressed concrete collar. The collar is clamped to the shaft using a prestressed system utilising continuous cable loops with in-line stressing anchors. A special procedure involving 600 tonne flat jacks in each leg position and final grouting of the collar/leg interface enables the connections to be made between the legs and the collar while the construction for the shaft progresses unhindered. The legs distribute some of the gravity load from the main shaft directly to the piles and also increase the resistance to overturning effects at the base of the tower.

There are three slots in the mid height region of the shaft to allow views from the lifts which are located inside the shaft. The slots are 92 m high by 1.6 m wide with 1.6 m deep coupling beams crossing at 8 m centres up the height of the slots to maintain shaft continuity.

The occupied levels in the upper section of the tower are referred to as the tower pod. The pod is divided structurally and by usage into three distinct zones; the lower pod, the mid pod and the upper pod. The lower pod zone contains space, for refuge (part of the emergency egress system) and communications. The mid pod region contains the public spaces, including a revolving brasserie floor and observation levels, and the upper pod contains further space for communications equipment.

The upper floors of the tower, outside the concrete shaft, are constructed using both composite structural steel-reinforced concrete and plain reinforced concrete. To avoid large cantilevers a series of hangers and struts support the floors in the mid pod region. Eight reinforced concrete (precast) fins which are stressed onto the shaft provide support for eight steel columns which

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in turn support the perimeter of the lower pod floors. The floors in the upper pod region, which are smaller in extent, cantilever from the concrete shaft.

The main concrete shaft terminates at the floor of the Sky Deck, the highest observation point open to the public, at 217.9 m above the street level. Above this level a structural steel framework, the pedestal, rises to support the upper lift machine room and a concrete ring slab which in turn supports the steel mast.

The mast is 90 m high and consists of five steel tubes with diameters ranging from 4.0 m to 0.4 m. These tubes are bolted together using gusseted flange connections and a perimeter ring of high strength bolts.

The top of the mast extends to 355.4 m above sea level and 326 m above street level making Sky Tower the seventh highest viewing and communication tower in the world.

DESIGN CONSIDERATIONS

Sky Tower has been designed to meet the requirements of current New Zealand design codes including the just released Concrete Structures Standard NZS 3101:1995.

The special nature of this structure has, however, required special consideration of a number of aspects including lateral wind loading, earthquake loading, detailing for seismic resistance and concrete durability. These aspects are discussed in the following sections.

DESIGN FOR WIND EFFECTS

By virtue of its height, the Sky Tower has the potential to be highly susceptible to wind load effects. This is in both terms of the maximum stresses to be expected in severe wind storms and the levels of acceleration to be expected during more moderate winds.

The tower has been designed to remain essentially undamaged when subjected to design wind speeds with an assessed return period of 1000 years. This design criterion is in excess of minimum code requirements for typical buildings in New Zealand (i.e. 350 years) but is considered warranted for a special structure of this type. It has been achieved by ensuring that the tower will have a dependable strength (as defined by the appropriate material design codes) in excess of that required to resist this level of loading. Under such conditions, the tower has been assumed to have a level of damping equal to 2% of critical.

For serviceability conditions, i.e. wind speed return periods of between as low as several weeks to five years, the tower has been checked to ensure that the accelerations at the habitable levels and the deflections at the communication levels are within acceptable criteria set to ensure adequate performance of the communication equipment and an acceptable level of occupant comfort. A level of damping in the tower shaft and mast equal to 0.8% and 0.3% of critical respectively was assumed for these low return period wind speeds.

A site specific study of the wind climate in which the available wind records from a number of sites in Auckland were investigated, was carried out by Auckland University (Auckland Uniservices Limited). This study confirmed that the extreme wind predictions of the New Zealand loading code were adequate for this structure but that the code provisions were likely to understate the frequency of occurrence of lower, serviceability winds.

Dynamic computer based analyses of the tower were carried out by the Boundary Layer Wind Tunnel Laboratory in Ontario, Canada. Computer analyses to establish wind loading response were carried out in preference to wind tunnel testing because of the difficulties in precisely modelling the response of tubular structures in the wind tunnel.

Analyses were performed to determine:

- maximum design actions in both along wind and across wind directions.

- the fatigue spectrum (relationship between stress amplitude and number of cycles) for the 100 years minimum fatigue life expectation for the tower and in particular the steel mast.

- accelerations in the habitable sections of the tower.

- accelerations over the height of the mast.

- the effect of additional damping (e.g. tunnel liquid mass dampers) to the mast to reduce inservice accelerations.

- the effects of vortex shedding on both the mast and tower.

A summary of the calculated ultimate limit state bending moments shear forces and displacements are shown on Figures 2, 3 and 4.

Wind tunnel tests are being completed at Monash University, Melbourne to assess pressures for the design of cladding elements.

SEISMIC DESIGN

The ramifications of a collapse of the Sky Tower would be catastrophic. It therefore demands a high level of protection against damage due to earthquake shaking notwithstanding the relatively low seismic hazard in Auckland City.

With input from the Institute of Geological and Nuclear Sciences (seismicity model) and overseas consultants Geomatrix Inc. (attenuation relationships for long period structures) a seismic hazard was completed for the site. The seismic hazard analysis defined the expected likelihood of given levels of seismic shaking in the tower and also defined the Maximum Credible Earthquake (MCE) for the project.

The tower shaft and foundations have been designed to remain essentially undamaged when subjected to the design earthquake motions defined in NZS 4203:1992 with an assigned nominal return period of 800 years. This has been achieved by designing these elements to have a dependable (reliable) strength at least that required by NZS 4203 taking μ (structural ductility factor) equal to 1.25 and R (risk factor equal to 1.2. Additional conservatism was provided by setting S_p = 1.0 (S_p is typically taken as 0.67 for normal buildings). A further factor of 1.3 was applied to the spectral values to reduce the implied level of damping for the tower shaft from 5% assumed in the code to a more realistic level of 2% for this structure and for this level of loading.

The tower shaft and foundations have also been designed to provide a high level of confidence of satisfactory performance during the MCE. This has been achieved by ensuring that these elements, when subjected to the MCE, will be either below their ideal capacity ($\emptyset = 1.0$) or will experience a level of ductility within that which can be reliably obtained given the level of detailing provided.

The magnitude and location of the MCE were assessed from the available geological data. The severity of the shaking was estimated, to a 95% confidence level, from the attenuation relationships derived during the seismic hazard analysis. A number of MCE scenarios were considered including a Richter magnitude 8.5 earthquake with an epicentre off East Cape and also smaller but closer events. The critical event was judged to be a Richter magnitude 7.0 event at 40 km distance from the site.

Analysis of the tower under the chosen MCE scenario showed that the shaft and foundations could resist the imposed actions without exceeding their ideal capacities.

The bending moments, shears and deformations calculated for the tower for the code, and MCE load cases described above are shown on Figures 2, 3 and 4 respectively. These are compared with the wind load values and also with the available ideal ($\emptyset = 1.0$) lateral capacity of the tower.

Notwithstanding that the MCE loading is the maximum level of earthquake shaking considered possible for the site, the designers felt that it was prudent to recognise the possible limitations in knowledge of the geology and tectonics of the area and consider even worse scenarios. The tower shaft and foundations have, therefore, also been detailed to ensure that in the event of greater levels of shaking, up to the maximum probable expected anywhere in New Zealand (i.e Richter magnitude 8 earthquake at 20 km), they will perform in a ductile manner without formation of any brittle mechanisms.

A lesser standard was required of the steel mast. The objective chosen was to resist code loads without damage (that is, within dependable strengths) and resist the MCE event without collapse. Using an inelastic time history analysis and an earthquake record derived to be consistent with the chosen MCE it has been shown that the only portion of the mast expected to yield in the MCE is the 400 mm diameter uppermost section. For this case the ductility demands are predicted to be well within the capacity of this tubular steel section.

CONCRETE DETAILING FOR DUCTILITY

To ensure non brittle behaviour in circumstances of overload, special detailing provisions have been incorporated. These include:

(a)   closely spaced stirrups to NZS 3101:1995 in critical regions of the shaft

(b)   diagonal reinforcement in the coupling beams crossing the shaft slots, and also between openings in the region of the pod.

(c)   adoption of capacity design procedures for determination of shear reinforcement requirements.

(d)   specification of f'_c = 45 MPa but with an upper limit at 28 days of 70 MPa.

CONCRETE DURABILITY

The concrete in all exposed areas of the shaft has been specified to include silica fume to achieve a high density, low permeability ($<1 \times 10^{-12}$ m/s), high

durability off-the-form finish. Trials indicated that silica fume added at the rate of 8% of cement content achieved the required finish and did not jeopodise workability. Superplasticiser was added to achieve the required workability and to ensure satisfactory placement with a final finish of uniform quality and colour.

SUMMARY

Design of New Zealand's tallest habitable structure, Sky Tower, has provided an interesting challenge to its designers and constructors. The tower configuration has some novel features compared with other similar towers already constructed around the world. These include the vertical slots in the concrete shaft and the raking legs to stiffen the tower.

Response to wind was a critical design issue and set the minimum level of stiffness for the tower. Special consideration was also necessary to give confidence of satisfactory seismic performance.

SOME SKYTOWER STATISTICS

Total Free Standing height
   above foundation   333.6 m

Total height above street level   325.9 m

Highest Public space (Sky Deck)   219.9 m
                       above street level

Materials to be used in construction.

   Volume of concrete   10,000 m³
   Quantity of Reinforcing Steel   1,400 t
   Structural Steel   660 t
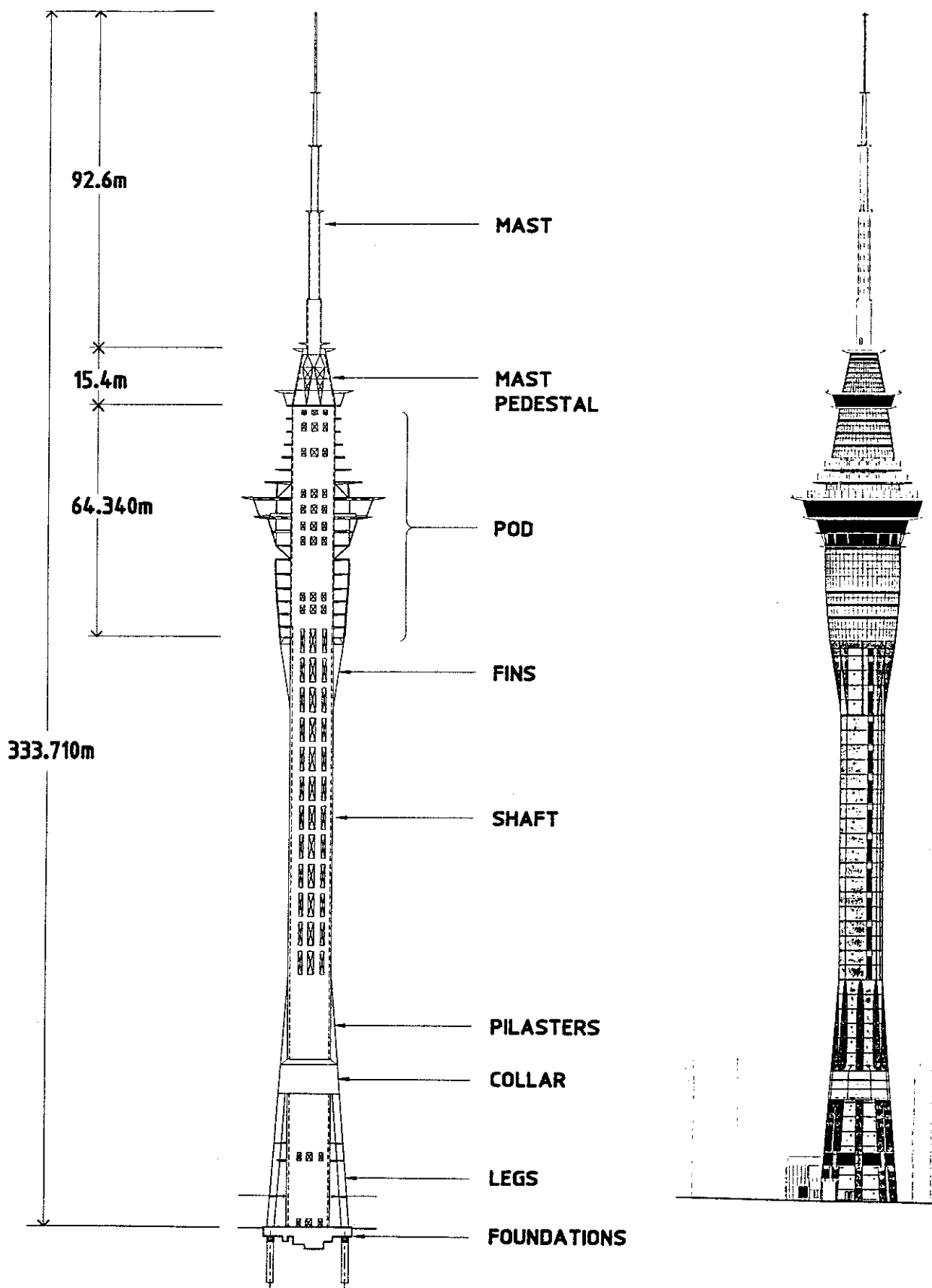
Construction Period   3 years

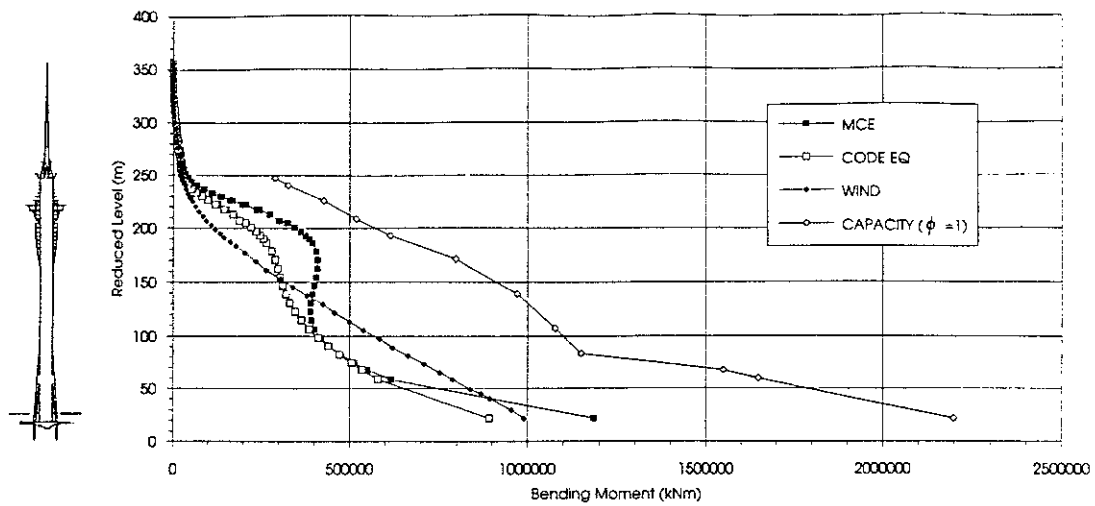Figure 1: Sky Tower Cross Section and Elevations

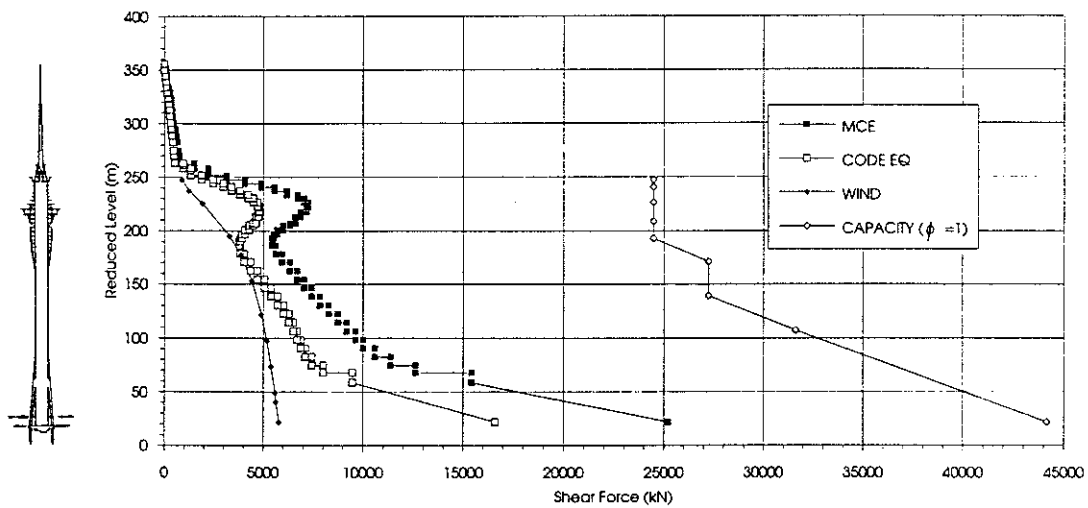Figure 2: Comparison of Bending Moment Requirements
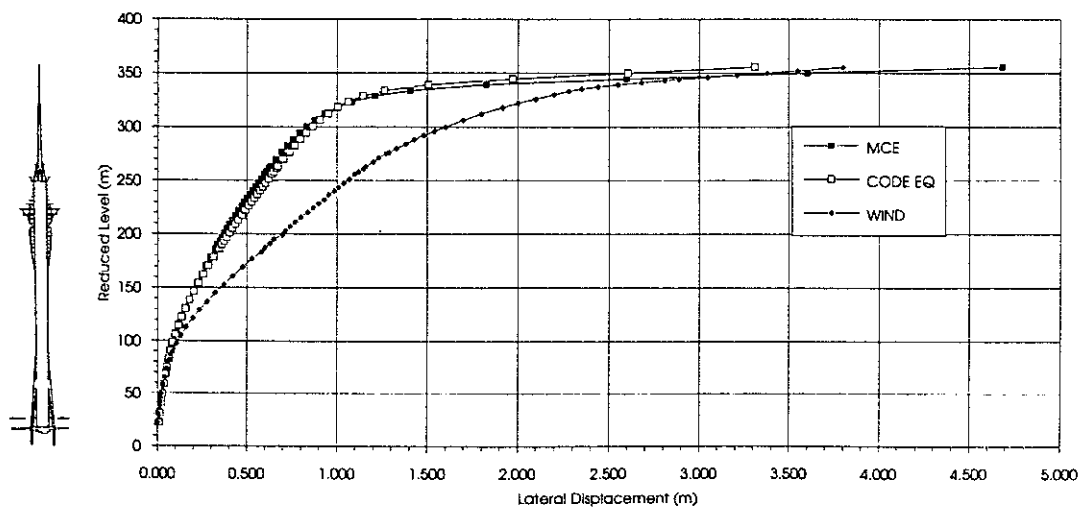

Figure 3: Comparison of Shear Force Requirements

Figure 4: Predictions of Lateral Displacement